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Testing and Evaluation of the McIntyre River Bridge Widening

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Project Research Engineer

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Abstract:

In this report a comparison is made between conventional bridge design analysis and test results of the McIntyre River Bridge. The factored resistence at U.L.S. of this rigid frame type bridge was established, and it was determined that traditional design methods are often conservative or even unrealistic.

Key Words: Rigid Frame, Skew, Slab, Thermal Loads, Live Loads

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1/ INTRODUCTION

This document describes load testing and performance evaluation for the widening of a bridge in Thunder Bay, District 19 over the McIntyre River (Figure 1).

During the planning stage of bridge testing, loads and their locations had to be selected so that the resulting strains would simulate the intensities of the specified AASHTO HS 20-44 design load plus impact. Load testing was provided by two MTC trucks. As the test loads were applied without dynamic effects, they had to be increased in a prescribed static manner. The strains and deflections resulting from this test indicated that a considerable safety margin was maintained even when the test load had reached its maximum intensity. To establish the load carrying capacities, it was therefore necessary to resort to analytical investigations. It is indicated in this report that analysis based on the "rigid frame" concept has resulted in much greater and different strains than those recorded during the testing. Several other analytical models were investigated, and it was concluded that the "fully restrained skew slab" concept compares with the test results in a more satisfactory manner than does the "rigid frame" concept.

The test has indicated the presence of high thermal strains. These have apparently resulted from a high degree of restraining of the deck slab and of the frame legs.

2/ DESCRIPTION OF BRIDGE

The McIntyre River Bridge is located 0.2 km west of John Street on the Thunder Bay Expressway, Hwy. 11 and 17. Bridge widening took place during late 1982 and early 1983. Load testing in May of 1983 was intended for the widened portion only, which at that time was closed to traffic and was still without asphalt paving.

Both the original bridge which is 13.6 m wide and the added portion of 9.2 m width are of an identical rigid frame type. The bridge has 25° skew angle and the clear skew span measures 15.25 m. The old and new bridge are interconnected along the junction of the deck by reinforcing steel dowels and are intended to perform as one bridge. Approaches are retained by 7.3 m long wing-walls and the embankments by the McIntyre River have 2:1 slopes covered with 0.6 m random riprap. Bridge footings are based on solid bedrock and at each end of the bridge deck a 6.0 m wide approach slab is attached. Figures 2 and 3 provide some of the details of the existing bridge and the widened portion.

Originally, the main steel reinforcement in the existing and the widened bridge portions had been intended to have almost identical layouts. Due to a construction error, however, some changes in reinforcing had to be made. As some of the negative moment reinforcing bars, extending from the rigid frame legs into the deck slab were mistakenly placed about 0.2 m lower than had been originally requested, additional bars had to be provided to cover the loss of effective negative steel. These bars were grouted into holes drilled into the concrete at the construction joint level. Additionally, positive reinforcing in the midspan of the deck slab was increased by about 30%. The essential details of the deck reinforcement and changes are indicated in Figure 6.

3/ INSTRUMENTATION OF BRIDGE DECK

Figure 5 shows the layout for instrumentation and identification of channels provided for bridge testing. Types of measuring devices are listed in Table 3.

In order to check the performance of some of the negative reinforcing steel, four lines of bars from the south half of the deck were instrumented with "Rosette" type strain gauges. Location of these is indicated in Figure 6.

The extent of deck rotation was observed with transducers No. 52 and 53 which were installed behind both the South and North abutment walls at the construction joint, level with the bottom of the deck slab. Demountable Strain Gauges (DSG) were installed on the top and bottom faces of the deck slab. Having gauges on the deck top introduced only minor restrictions on the test vehicle movements. Deck slab deflections were measured with Linear Voltage Displacement Transducers (LVDT). Due to a shortage of available channels at the time, only the south half of the deck was instrumented. All of the measuring devices were electronically connected to the mobile lab truck containing the data acquisition and recording equipment.

4/ TEST LOADS AND THE LOADING SEQUENCE

Two MTC test vehicles, identified in this report as K and M, are the Kenworth truck and King trailer and the Mack truck and Davis trailer respectively. Table 2 indicates basic dimensions and load distribution.

The selected positions of these vehicles on the bridge during the test are indicated in Figure 4. Positions are identified as K(1); [K(1) + M(1)]; K(2); [K(2) + M(2)]. The numbers (1) and (2) indicate which lane the vehicles occupied. The loading in lane (1) provided typical bridge performance data along the free exterior edge of deck. The loading in lane (2) provided performance data for the interior portions of the deck which was expected to have much less skew effect than the exterior positions along the free edge. The position chosen for the rear axle of the testing trucks is indicated as 3.65 m and was intended to provide maximum response for the negative moment reinforcing steel. Positive moment areas were considered to be safe and therefore of lesser interest in testing.

The test loading on each of the vehicles started with 24 "blocks". This load will be identified as Load Level 1. The load was then gradually increased up to a maximum load of 72 blocks on each of the vehicles and is identified as Load Level 5. Sequence of loading is provided in the Table 2.

Recording of test data was done as follows: readings were taken before and after the Kenworth truck (K), and the Kenworth and Mack (K + M) trucks in combination were moved into the designated loading positions. They provided a "zero" reference for the intermediate readings of the loaded bridge response. As an indication of test accuracy, Figure 11 shows the typical drift pattern of the zero readings.

5/ SUMMARY OF TEST RESULTS

A brief summary of test data for Load Level 5 will be presented. Some measured bridge deflections are indicated in Table 4. During the maximum load of 72 blocks none of the deflections exceeded 1.5 mm. These measurements were taken by a Lineal Voltage Displacement Transducers (LVDT). Some of the displacements were confirmed with a precision level.

Table 5 provides tensile and compressive concrete strains measured by the DSG. None of the tensile strains exceeded 120 $\mu\varepsilon$ (micro strains = 10^{-6}). As strains of this magnitude were below the tensile crack formation limits, and the bridge was loaded to this level for the first time, they were considered to be of homogeneous concrete material.

Table 6 indicates tension and compression strains as recorded at the top of the parapet wall. A few of these strains are slightly above the tensile strength of concrete (0.1×10^{-3}).

Strains in the reinforcing bars (No. 8, 14 and 28) are shown in Table 7. When at the maximum load of 72 blocks these strains were similar to those recorded at the identical deck locations at the top of concrete. This performance feature indicated proper bonding existed between the reinforcing steel and concrete. Transducers No. 52 and 53 showed only slight deck rotations during the loading.

In all cases, starting from a 24 block load, proportionality of load and corresponding strain was maintained. At no time was a distress situation apparent or indicated.

Plots of strain distribution prove the presence of negative and positive moments in the deck slab as is illustrated in Figures 7, 8 and 9. From so-called zero readings (before and after the bridge loading) the thermal strains have been plotted in Figure 10. Despite the considerable "drift" as indicated in Figure 11, and presumed lack of accuracy, the observation can be made that the thermal restraints are of a much higher order than those which could have been estimated for a "rigid frame". In addition,

these thermal strains were even higher than those established for the maximum test Load Level 5. Temperature recorded at the top of deck was 9°C higher than at the bottom. Figure 10 is a record of thermal strains at 9:30 a.m. and 12:30 p.m.

6/ ANALYTICAL INTERPRETATION OF THE TEST DATA

Analytically, performance of a skew slab may be investigated using computer aided design, such as the "finite difference" or "finite element" methods. Where highly sophisticated information is not required, other available design aids like influence surface charts are adequate to establish moments and shears at the critical deck slab locations. This latter method was undertaken during the investigation of this bridge. For the skew slabs tested, the centre of slab, the obtuse corner, and close to the centre of the skew span along the free edge were the critical points evaluated. The edge stiffening provided by the curb and parapet wall resulted in the load being attracted toward this edge and caused larger transverse moments than in an unstiffened slab.

The restraining longitudinal negative moment at the free edge in the obtuse corner was estimated to be up to three times as large as a negative moment in the central portion of the same slab. At the same time the positive moment along the free edge was only twice as large.

It is well known that the most important factors affecting the skew slab performance are the skew angle, ratio of span-to-width and type of slab supports. The bridge has a span-to-width ratio of about 0.7. For this ratio, analysis for central deck portion was done without investigating the skew effects. It was proven that the analysis results were satisfactorily close to those recorded during the testing.

The bridge designer followed the conventional simplified design methods. He assumed that the bridge performed as a rigid frame acting along the skew span. The live load of one line of the design truck's wheels was distributed within 2.0 m width of the deck as prescribed by AASHTO Code. Yet, current analytical investigation has shown that the 25° skew tends to override the above assumed classic performance for a rigid frame. The departure was noted when evaluating and comparing the results of analysis with the obtained test data. Full restraint of the deck slab was therefore assumed and proven to be justified in the investigation of live load and thermal effects. For dead load moments, however, predictions

were made using the rigid frame concept. In all of the analytical models investigated, the slab was assumed to perform as:

A/ fully restrained beam acting along the skew span;

B/ fully restrained slab investigated as a "full slab strip" using square span; and

C/ fully restrained slab investigated as a "half slab strip" using skew span.

Together, these analytical investigations indicate much higher to moderately higher strains than those actually obtained during the bridge testing. Closest to the test results are the predictions based on Models B and C. Model B is apparently adequate to simulate the responses of the interior portions of the deck and Model C the exterior portions. Tables 8 and 9 provide the summary of the discussed investigations. In these tables, in addition to the analytical values for the test trucks, the moments and strains for AASHTO and OHBD Code design trucks have been estimated.

Finally, evaluation of thermal performance of the bridge was attempted. Strain predictions assuming the slab to perform as a shallow restrained arch provided the best correlation with the test. Table 10 gives information to be compared with that in Figure 10.

7/ LOAD FACTORS AND MAXIMUM FLEXURAL RESISTANCE

At ultimate limit states (ULS) the Limit States Code permits factoring of the moments which have been derived, assuming the slab is fully isotropic material. Such an assumption may only be correct when concrete compression strains are still below the 2×10^{-3} value, and in this state the slab is still within the fully isotrophic and elastic serviceability range. At ULS the localized yielding and subsequent stress redistribution may take place justifying to a certain extent the current code practice for load factoring.

To establish the maximum flexural effects at the critical points of this bridge, the unfactored load moments established for the serviceability states have been projected into ULS applying the appropriate load factors and performance factors as specified by AASHTO and OHBD Codes. As mentioned before, the serviceability dead load moments were established using the "rigid frame" concept, and the "fully restrained slab" concept to establish the applied live load moments. The factored flexural resistance has been expressed as moment (m_x), where;

$$m_x = F\, M_{LL} = \frac{\left[\phi R_n - \alpha_{DL} M_{DL}\right]}{\alpha_{LL}}$$

Estimated resistance moments (m_x) in Table 11 provides a direct means of comparing these with the unfactored moments listed in Table 8. It is indicated that the Factor (F) in Table 11 is always larger than one and is proof that safety margins are more than adequate using the analysis discussed. Furthermore, a considerably larger safety margin is to be expected with more refined analysis.

8/ CONCLUSIONS

It has been demonstrated that the widened portion of the McIntyre Bridge is adequate for the design loads HS 20-44 as specified in the Figures. The standard rigid frame design method has yielded highly conservative and often unrealistic results. Methods based on slab theory and assuming full restraint have demonstrated better agreement with the test data.

The use of the refined investigation methods based on the isotropic slab theory may not always be justified in the performance predictions for a skew slab at ULS. The yielding of the skew slab at ULS is highly localized and is more pronounced than for square slabs. Slab at failure loads, due to earlier cracking, tends to perform as a number of separate beams interconnected by transverse reinforcement. The methods based on yield line theory developed for rectangular slabs should therefore not be used for slabs with a high degree of skew.

Table 1/ General Arrangement of Bridge

NOTES:

Reinforcing Steel Grade ---------- 400

Bars marked with Suffix 'C' are coated.

Class of Concrete:

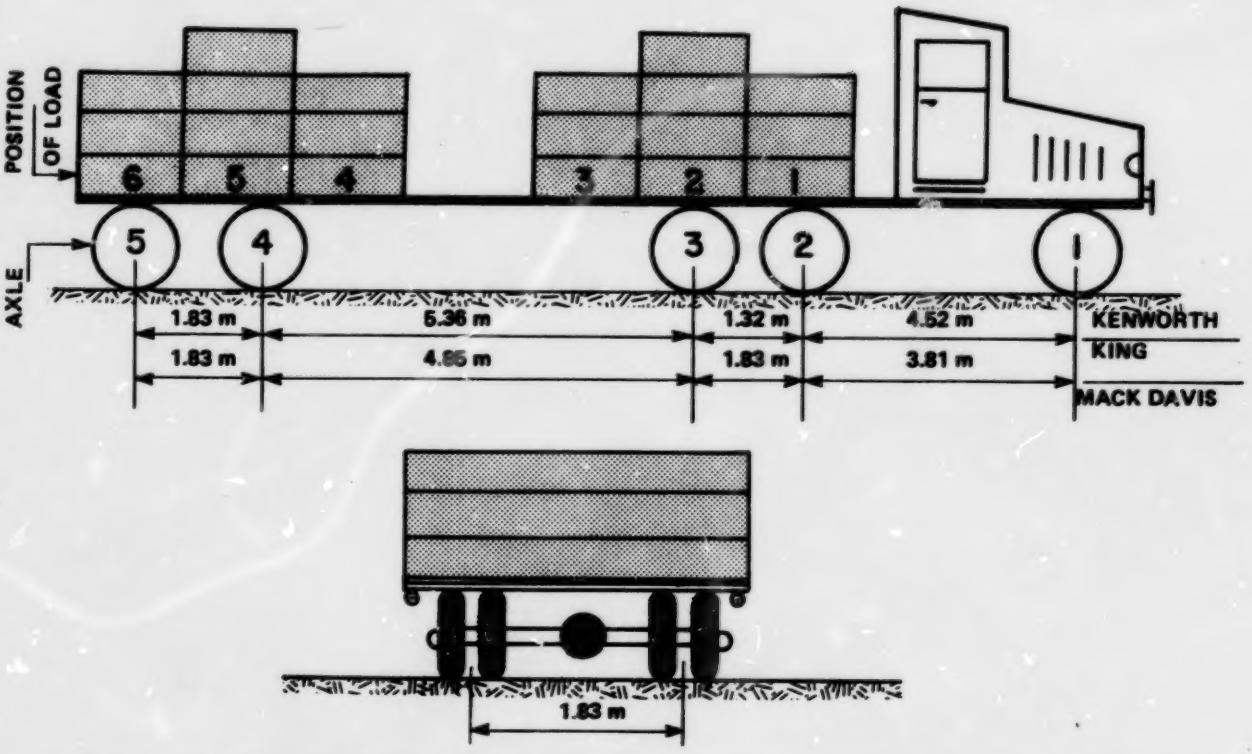
Deck and Parapet Walls ---------- 30 MPa

Remainder and/or as noted on drawings --------- 20 MPa

Clear Cover of Reinforcing Steel:

Footings -- 75 mm,
Abutment walls -- 50 mm,
Deck Top -- 50 mm,
U/S -- 35 mm and/or as noted on drawings.

Table 2/ Test Vehicles with Various Load Levels



	LOAD LEVEL	BLOCKS	DISTRIBUTION OF BLOCKS IN POSITION 6	5	4	3	2	1	WEIGHT PER AXLE [kN] 5	4	4	2	1	TOTAL [kN]
KENWORTH (K)	1	24	4	4	4	4	4	4	105.0	105.0	92.6	92.6	47.6	442.8
	2	36	8	6	4	4	6	8	136.6	136.6	115.2	115.2	49.4	553.0
	3	48	8	8	8	8	8	8	168.7	168.7	138.4	138.4	51.6	665.8
	4	60	12	10	8	8	10	12	200.7	200.7	161.5	161.5	53.4	777.8
	5	72	12	13	10	10	14	12	231.8	231.8	185.1	185.1	55.9	889.7
			TARE WEIGHT [kN]						40.9	40.9	46.3	46.3	43.6	218.0
MACK (M)	1	24	4	4	4	4	4	4	104.6	104.6	89.9	89.9	43.6	432.6
	2	36	8	6	4	4	6	8	137.5	137.5	113.0	113.0	44.5	545.5
	3	48	8	8	8	8	8	8	170.0	170.0	135.7	135.7	45.8	657.2
	4	60	12	10	8	8	10	12	202.9	202.9	158.9	158.9	46.7	770.3
	5	72	12	14	10	11	13	12	236.3	236.3	181.1	181.1	47.6	882.4
			TARE WEIGHT [kN]						39.2	39.2	44.0	44.0	41.4	207.8

Table 3/ Instrumentation

INSTRUMENT	SYMBOL
Demountable Strain Gauges (DSG) Index: 10 to 25, 50, 51 and 54 Unit of Measure: ($\mu\varepsilon = 10^{-6}$)	
Lineal Voltage Displacement Transducer (LVDT) Index: 52, 53, 55 to 66 Unit of Measure: (1/1000 in = 1/25 400 mm)	
Thermocouple Index: T0, T7, T5 and T6 Unit of Measure: Celcius	(T)
Rosette Strain Gauges (on reinforcing bars) Index: 70 to 77 and 80 to 87 Unit of Measure: ($\mu\varepsilon = 10^{-6}$)	

Table 4/ Deflections of Deck at Load Level 5 ($\Delta \times 10^{-3}$ mm)

See Figure 5 for Index of Channels

	LOAD IN LANE 1		LOAD IN LANE 2		
CHANNEL INDEX	K(1) POSITION 1	(K+M)(1) POSITION 2	K(2) POSITION 1	(K+M)(2) POSITION 2	
57	+ 17	+ 125	+ 71	+ 368	
60	+ 48	+ 356	+ 265	+ 549	
63	+ 25	- 46	- 6	- 46	
66	+ 315	+ 311	+ 71	+ 47	
56	+ 121	+ 792	+ 512	+1372	
59	+ 242	+ 931	+ 493	+1349	
62	+ 735	+1464	+ 960	+1398	
65	+1025	+1438	+ 652	+ 786	
55	+ 18	+ 146	+ 79	+ 390	+ down
58	+ 18	+ 260	+ 193	+ 493	- up
61	+ 223	+ 510	+ 429	+ 586	
64	+ 539	+ 629	+ 295	+ 340	
52	+ 6	+ 6	+ 6	- 11	
53	- 18	- 6	0	- 12	

Table 5/ Concrete Strains of Deck Slab ($\mu\varepsilon$)
See Figure 5 for Index of Channels

At Load Level 5 -- + Tension, - Compression

CHANNEL INDEX	LOAD IN LANE 1				LOAD IN LANE 2			
TOP / BOTTOM	POSITION 1 K(1)		POSITION 2 (K+M)(1)		POSITION 1 K(2)		POSITION 2 (K+M)(2)	
	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM
13 / 33	+ 1	0	+10	-11	- 4	- 9	+13	-25
17 / 37	+10	?	+21	-15	+ 6	-16	+28	-12
21 / 41	+ 8	-17	+29	-21	+14	-25	+35	-19
25 / 45	+18	-21	+32	-32	+ 5	-22	+32	-10
29 / 49	?	-30	+29	-24	+ 8	-17	+24	- 6
12 / 32	- 1	+ 3	+ 2	+ 7	- 1	- 4	-12	+ 1
16 / 36	- 2	?	- 6	+ 3	- 6	-14	+ 9	- 4
20 / 40	-12	+ 4	+ 2	-16	?	-23	+10	-26
24 / 46	+ 1	+12	+ 6	- 8	+ 3	-15	+20	- 1
28 / 48	+11	+ 4	+ 6	- 1	- 1	- 4	+18	0

Table 5/ Concrete Strains of the Deck Slab (με) (continued)
See Figure 5 for Index of Channels

+Tension, -Compression

CHANNEL INDEX	LOAD IN LANE 1				LOAD IN LANE 2			
TOP / BOTTOM	POSITION 1 K(1)		POSITION 2 (K+M)(1)		POSITION 1 K(2)		POSITION 2 (K+M)(2)	
	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM
11 / 31	- 1	+ 3	- 6	+12	- 6	- 4	-36	+25
15 / 35	- 8	0	-31	+11	-20	- 5	-30	+13
19 / 39	-10	+ 6	-19	+22	-14	+ 6	-28	+23
23 / 43	-11	+50	- 8	+119	-21	+72	-10	+87
27 / 47	- 5	+ 6	- 9	+20	- 3	- 5	+ 8	+ 8
10 / 30	-11	+ 8	-32	+17	-61	+ 2	-52	+32
14 / 34	- 7	0	-29	+ 8	-11	- 3	-18	+ 9
18 / 38	-17	+ 8	-16	+18	- 9	- 1	-27	+18
22 / 32	-13	+20	-32	+43	-15	+20	-16	+40
26 / 46	- 2	+26	- 8	+18	-18	+13	+11	+15

Table 6/ Strains on Surface of Parapet Wall at Load Level 5 (με)
See Figure 5 for Index of Channels

+Tension, -Compression

CHANNEL INDEX	LOAD IN LANE 1				LOAD IN LANE 2			
TOP / BOTTOM	POSITION 1 K(1)		POSITION 2 (K+M)(1)		POSITION 1 K(2)		POSITION 2 (K+M)(2)	
	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM	TOP	BOTTOM
51 49	+21	-30	+24	-24	- 2	-17	+23	- 6
50 46	- 5	+26	-31	+18	-20	+13	0.0	+15
54	+15	-	+ 4	-	- 2	-	+ 5	-
55	+144	-	+64	-	+192	-	+96	-

Table 7/ Strains on Reinforcing Bars at Load Level 5 ($\mu\varepsilon$)
See Figure 6 for the Bar Layout

+ Tension, - Compression

		LOAD IN LANE 1		LOAD IN LANE 2	
BAR INDEX	CHANNEL INDEX	POSITION 1 K(1)	POSITION 2 (K+M)(1)	POSITION 1 K(2)	POSITION 2 (K+M)(2)
	80	--	--	--	--
	81	- 6	+10	- 4	+ 5
	82	-11	+17	-11	+12
	83	-11	+19	- 8	+12
No. 8	84	- 5	+ 8	- 6	+ 7
	85	+ 6	- 3	- 1	+ 2
	86	+12	-14	+ 7	- 6
	87	+ 5	- 9	+ 7	- 8
	70	- 6	+14	- 7	+ 2
	71	- 3	+11	- 7	+10
	72	- 7	+20	-12	+20
	73	- 2	+16	-10	+20
No. 28	74	+ 3	+ 3	+ 2	+ 4
	75	+ 6	-13	+10	-20
	76	+ 6	-22	+13	-20
	77	+ 8	-17	+10	-20
	78	+ 5	+11	+ 6	+ 9
No. 14	79	+ 4	+10	+ 7	+ 9

Table 8/ Unfactored Live Load Moments Per Unit Width of Slab [kN m/m]

The moments indicated have been calculated in accordance with the following:

A -- Beam theory, applying Code specified load distribution factors for the skew span.

B -- "Full slab strip" theory applied to the right span.

C -- "Half slab strip" theory applied to the skew span.

AASHTO and OHBDC Truck Load Moments have included the Dynamic Load Allowance and factors to account for the multi-presense of trucks as specified by the respective Codes.

			ONE TRUCK			TWO TRUCKS		
CASE	METHOD OF ANALYSIS	m_x^o	AASHTO TRUCK	OHBDC TRUCK	TEST TRUCK	AASHTO TRUCKS	OHBDC TRUCKS	TEST TRUCK
A	Beam-Skew Span	1.0	-171	-313	-372	-171	-274	-325
		0.5	+106	+190	+212	+106	+166	+190
B	Slab-Right Span "Full Strip"	1.0	- 77	-106	-133	-132	-156	-197
		0.5	+ 32	+ 59	+ 67	+ 54	+ 93	+116
C	Slab-Skew Span "Half Strip"	1.0	-254	-376	-425	-436	-553	-629
		0.5	+ 79	+152	+163	+135	+239	+280

Table 9/ Theoretical Concrete Strains ($\mu\varepsilon$)
Assuming uncracked sections and moments shown in Table 8

+ Tension, - Compression

			ONE TRUCK			TWO TRUCKS			
	METHOD OF ANALYSIS	POINT X/L	AASHTO TO TRUCK	OHBDC TRUCK	TEST TRUCK	AASHTO TRUCKS	OHBDC TRUCKS	TEST TRUCKS	REMARKS
A	Beam-Skew Span	1.0	±31	± 56	± 67	±31	± 49	± 58	Top or Bottom
		0.5	∓86	∓154	∓173	∓86	∓135	∓154	"
B	Slab-Right Span	1.0	±14	± 19	± 24	±24	± 28	± 35	"
		0.5	∓26	∓ 48	∓ 54	∓44	∓ 76	∓ 94	"
C	Slab-Skew Span	1.0	+51	+ 77	+ 90	+57	+ 84	+ 99	Top of Barrier
			+14	+ 21	+ 25	+16	+ 23	+ 27	Top of Curb
			±11	± 16	± 19	±12	± 17	± 21	Top or Bottom of Slab
		0.5	-36	- 54	- 61	-36	- 56	- 62	Top of Barrier
			- 7	- 10	- 11	- 7	- 10	- 12	Top of Curb
			∓ 4	∓ 6	∓ 7	∓ 4	∓ 6	∓ 7	Top or Bottom of Slab

Table 10/ Theoretical Thermal Strains ($\mu\varepsilon$)
Deck is performing as a restrained flat arch: top 16°C, bottom 7°C

+ Tension, - Compression

PT	$X_{(m)}$	$\mu\varepsilon$	
		Top	Bottom
1	0.61	-173	+ 30
2	3.05	-179	+ 21
3	5.49	-148	+110
4	7.62	-126	+155

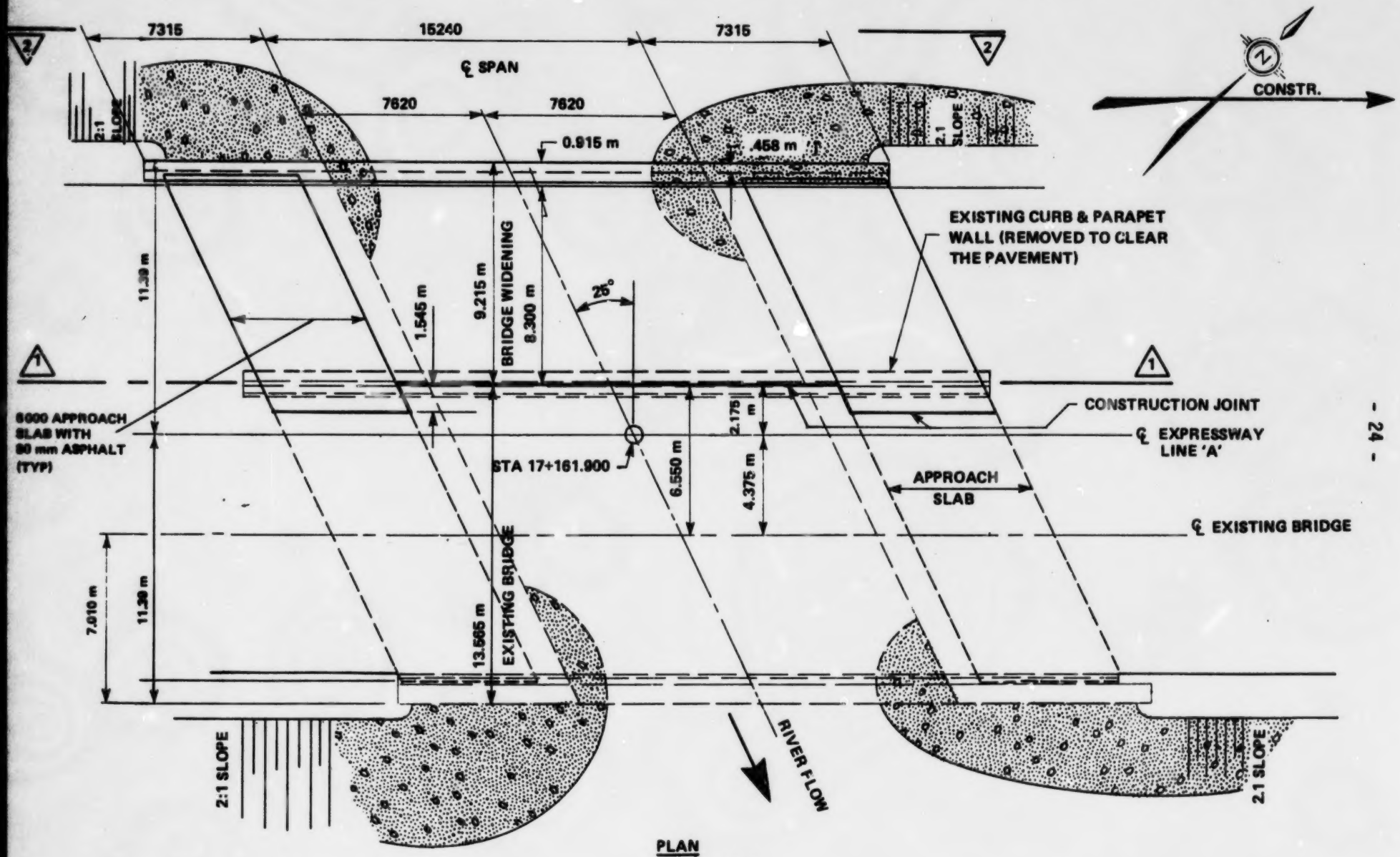
Note: $X_{(m)}$ signifies the distance from the face of abutment in metres measured along the skew span.

Table 11/ Factored Flexural Resistance for Live Load [kN m/m]
Based on the provisions of AASHTO and OHBDC

$$m_x = FM_{LL} = \frac{\left[\phi R_n - \alpha_{DL} M_{DL}\right]}{\alpha_{LL}}$$

Area of Application	m_x	AASHTO	OHBDC
"Interior Strip" Right Span	m_1	-472	-753
	$m_{0.5}$	+226	+357
"Exterior Strip" Skew Span	m_1	-385	-626
	$m_{0.5}$	+306	+486

Figure 1/ Bridge Testing Site



PLAN
SCALE 1: 200

Figure 2/ McIntyre River Bridge Plan

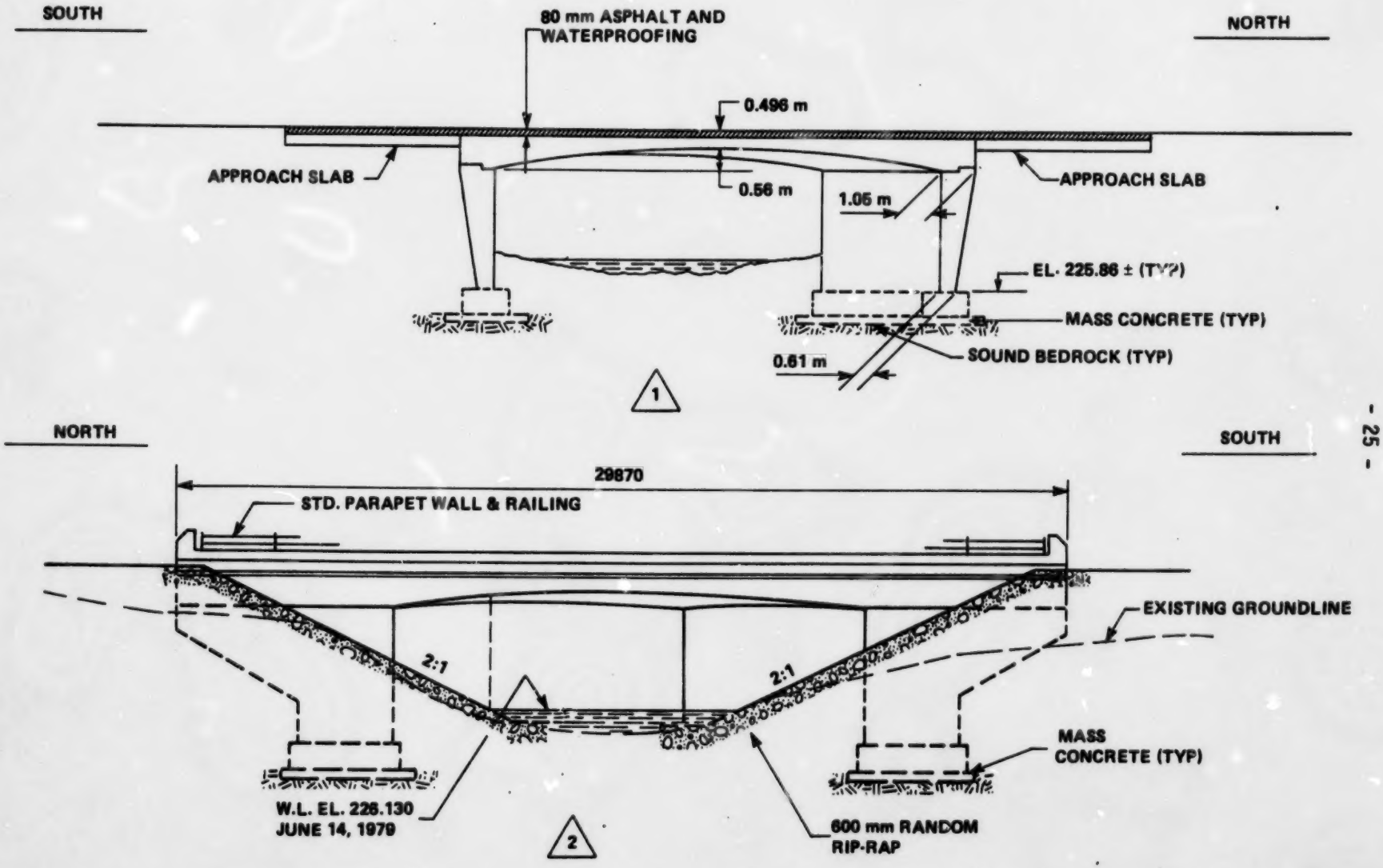


Figure 3/ McIntyre River Bridge Sections

NOT TO SCALE

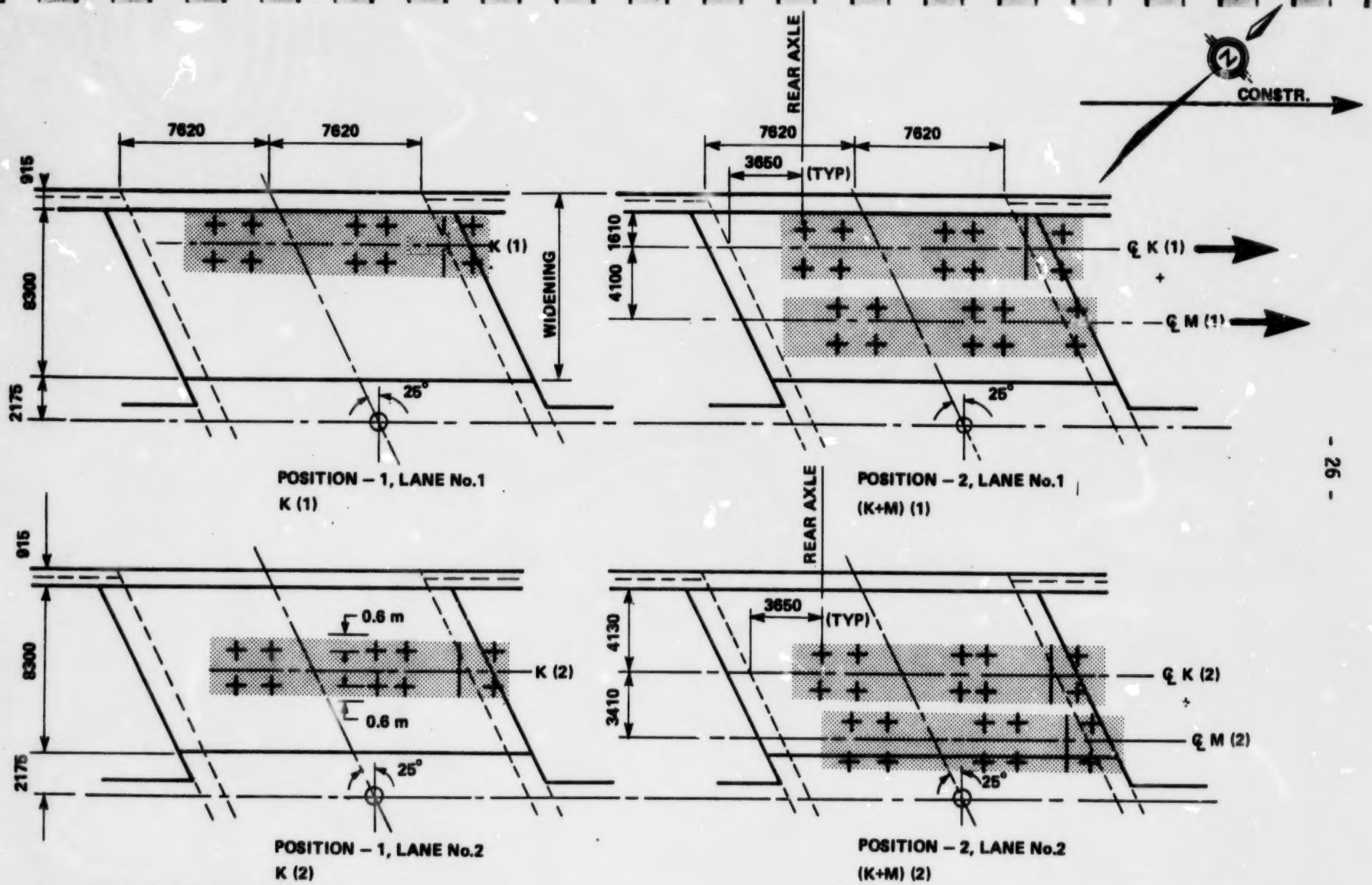


Figure 4/ Test Vehicle Positions

NOT TO SCALE

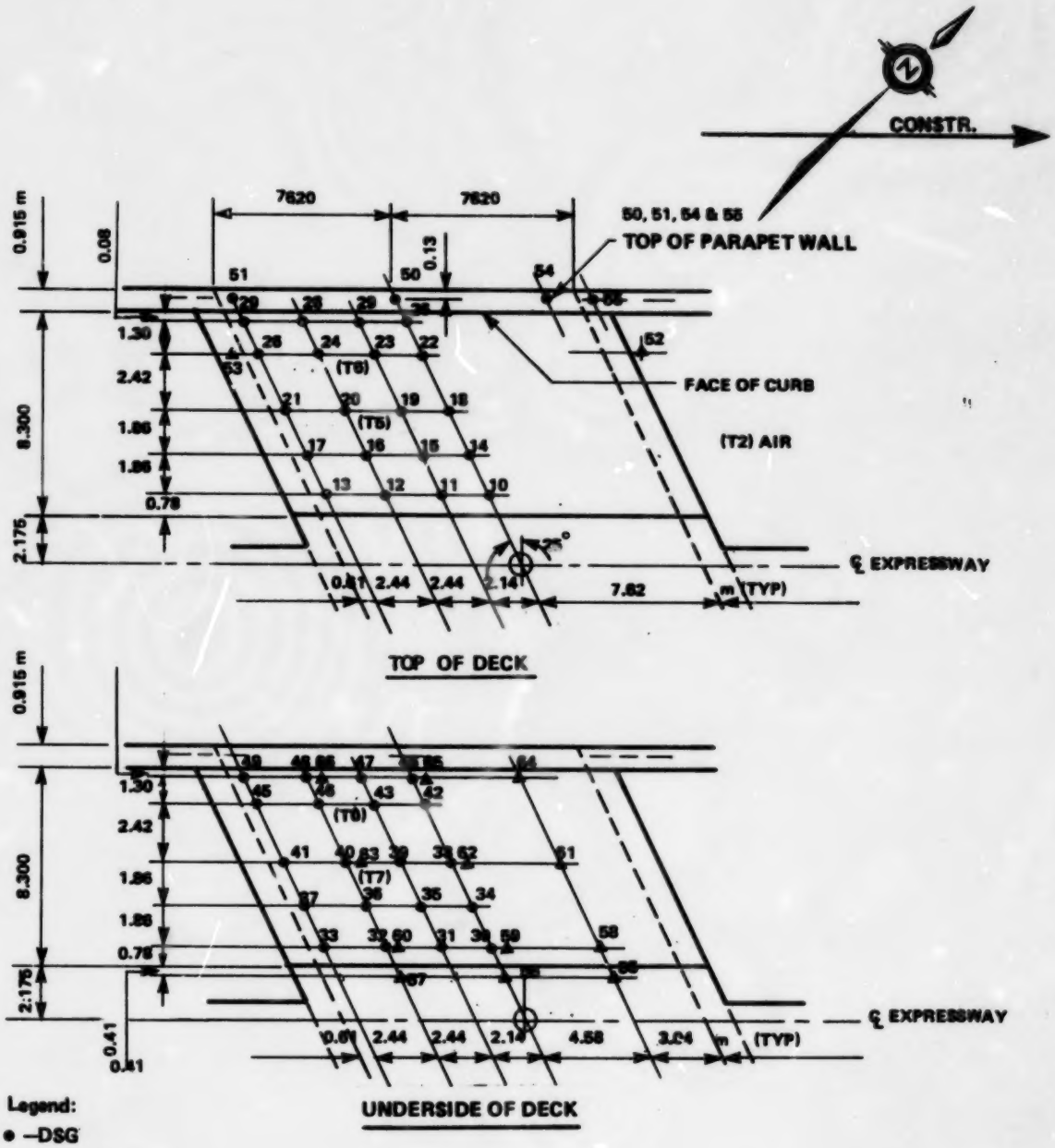


Figure 5/ Instrumentation Layout and Index of Channels

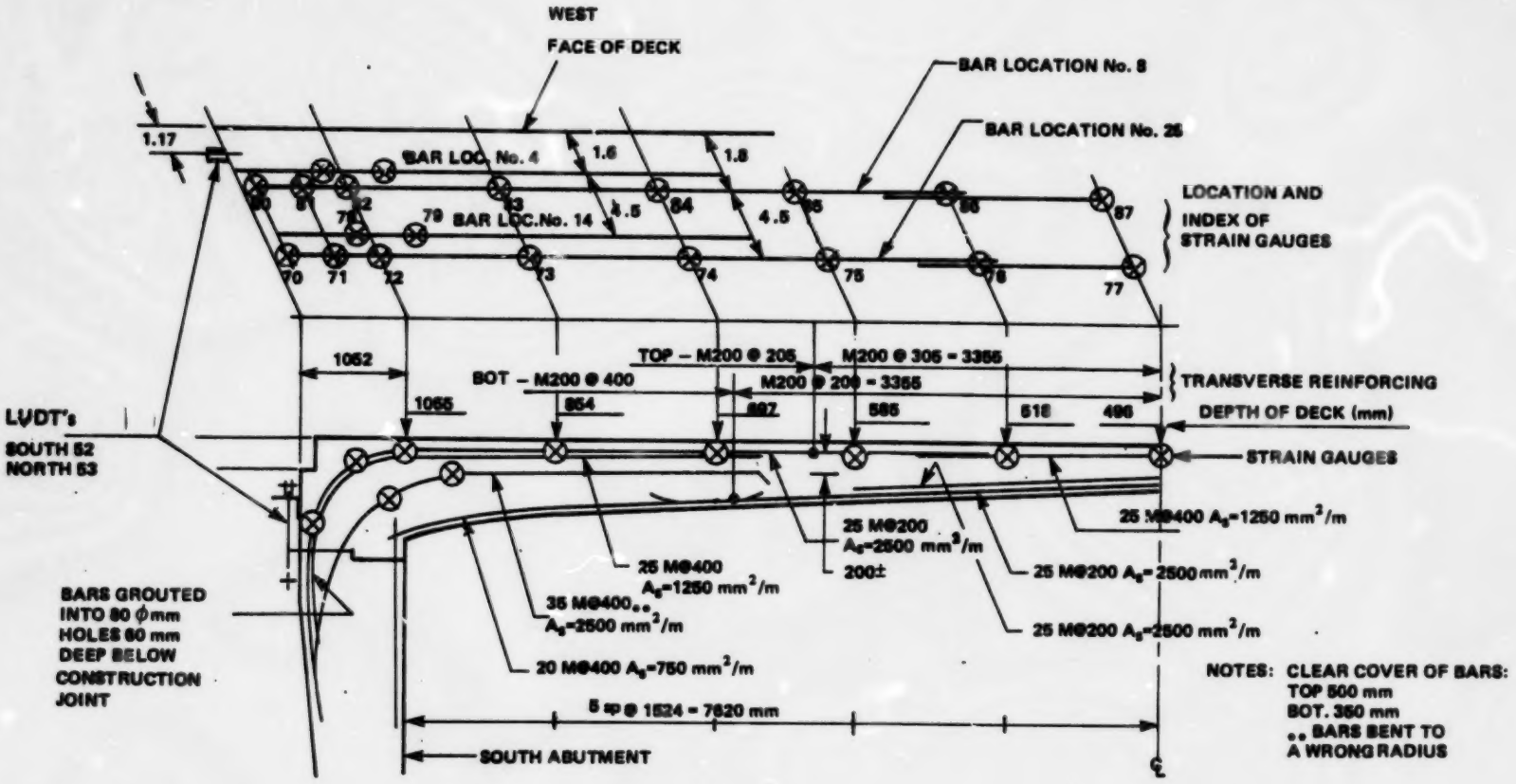


Figure 6/ Typical Layout of Reinforcing Steel

Note: Location of strain gauges mounted on reinforcing bars is indicated by ⊗ (South only).

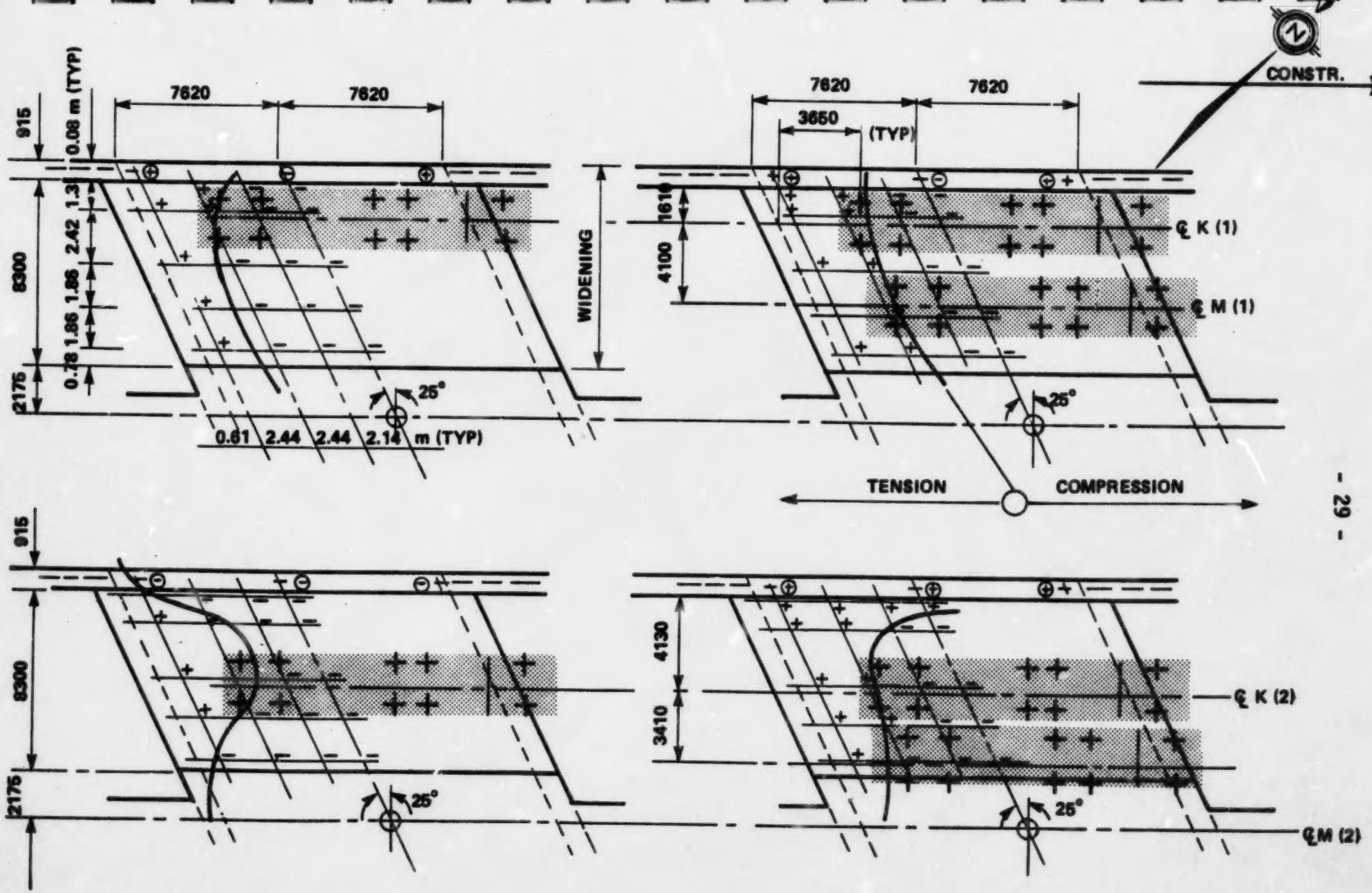


Figure 7/ Tension and Compression Areas of Top of Deck
Based on Concrete Strains in Table 5

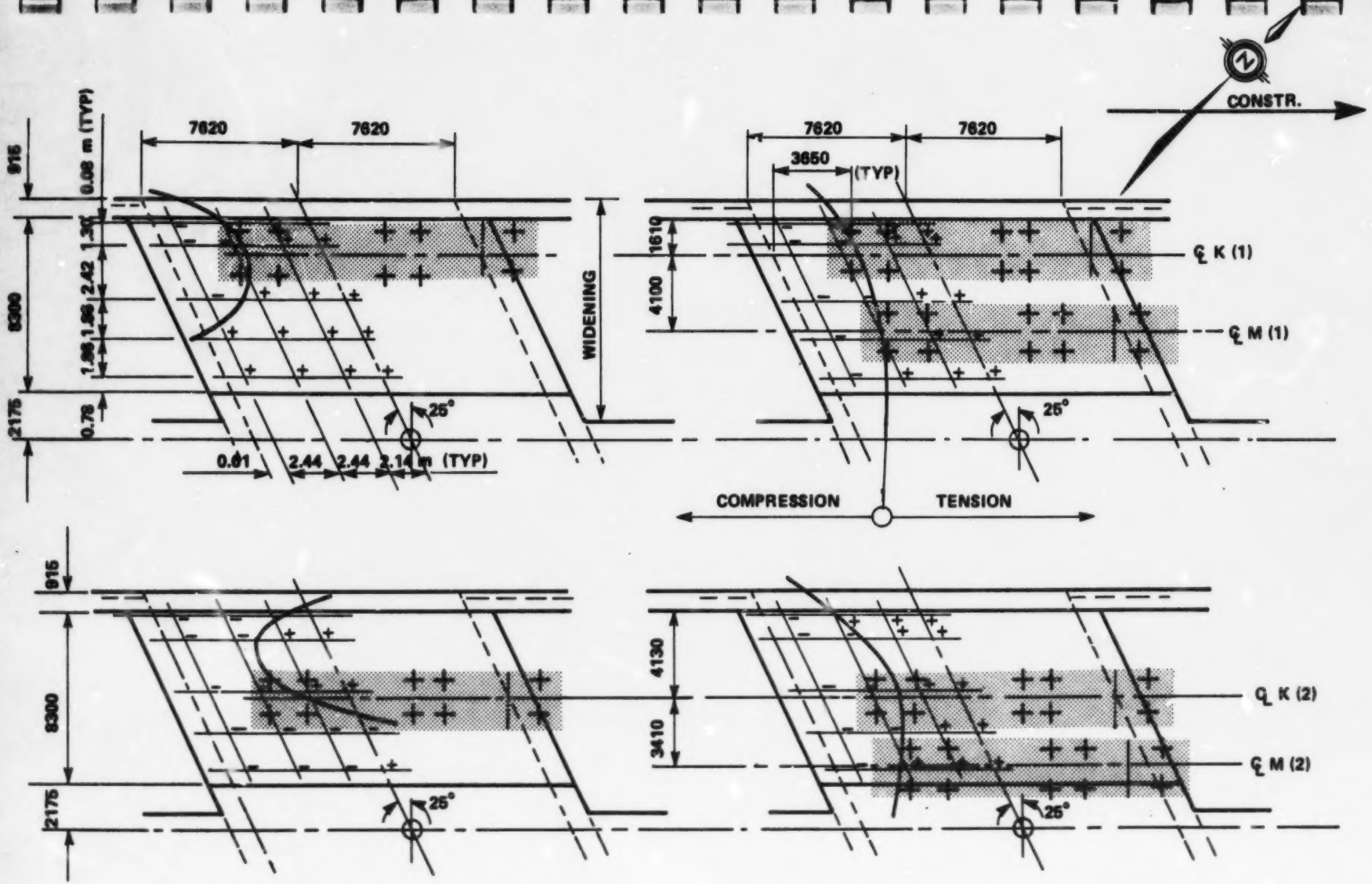


Figure 8/ Tension and Compression Areas of Underside of Deck
Based on Concrete Strains in Table 5

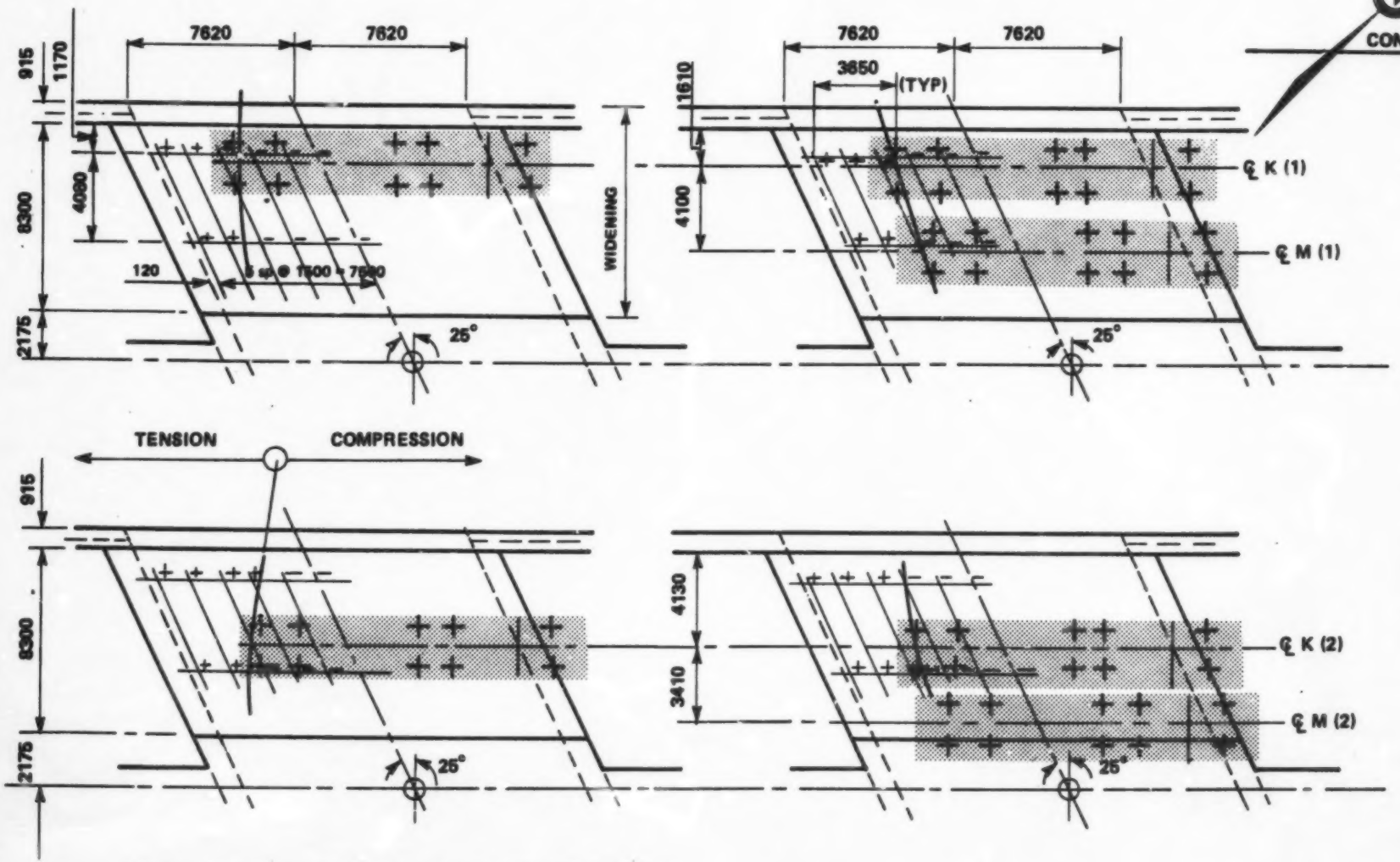


Figure 9/ Tension and Compression Areas of Top of Deck
Based on Reinforcing Steel Strains in Table 7

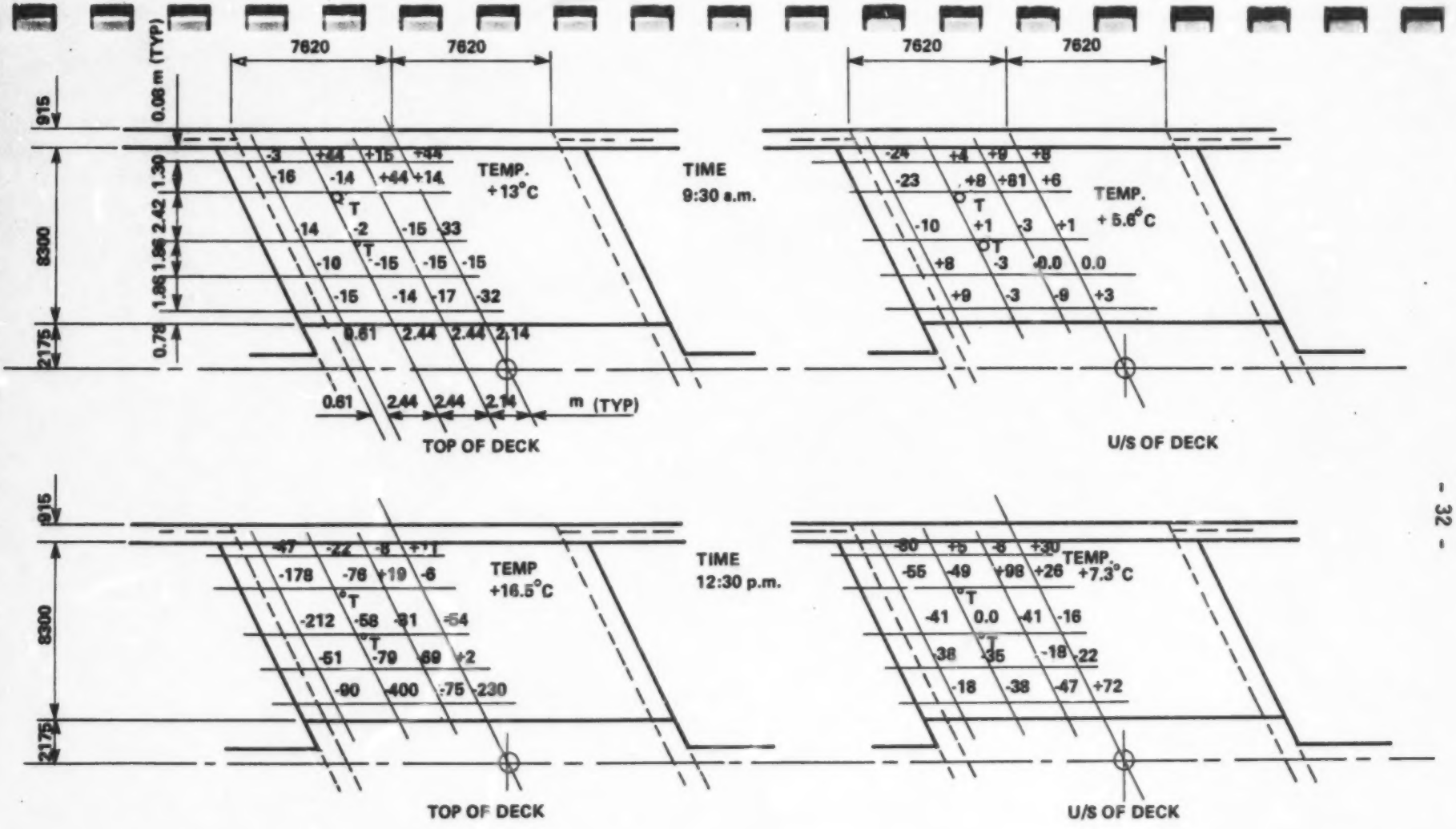


Figure 10/ Temperature Strains ($\mu\epsilon$)
- Compression
+ Tension

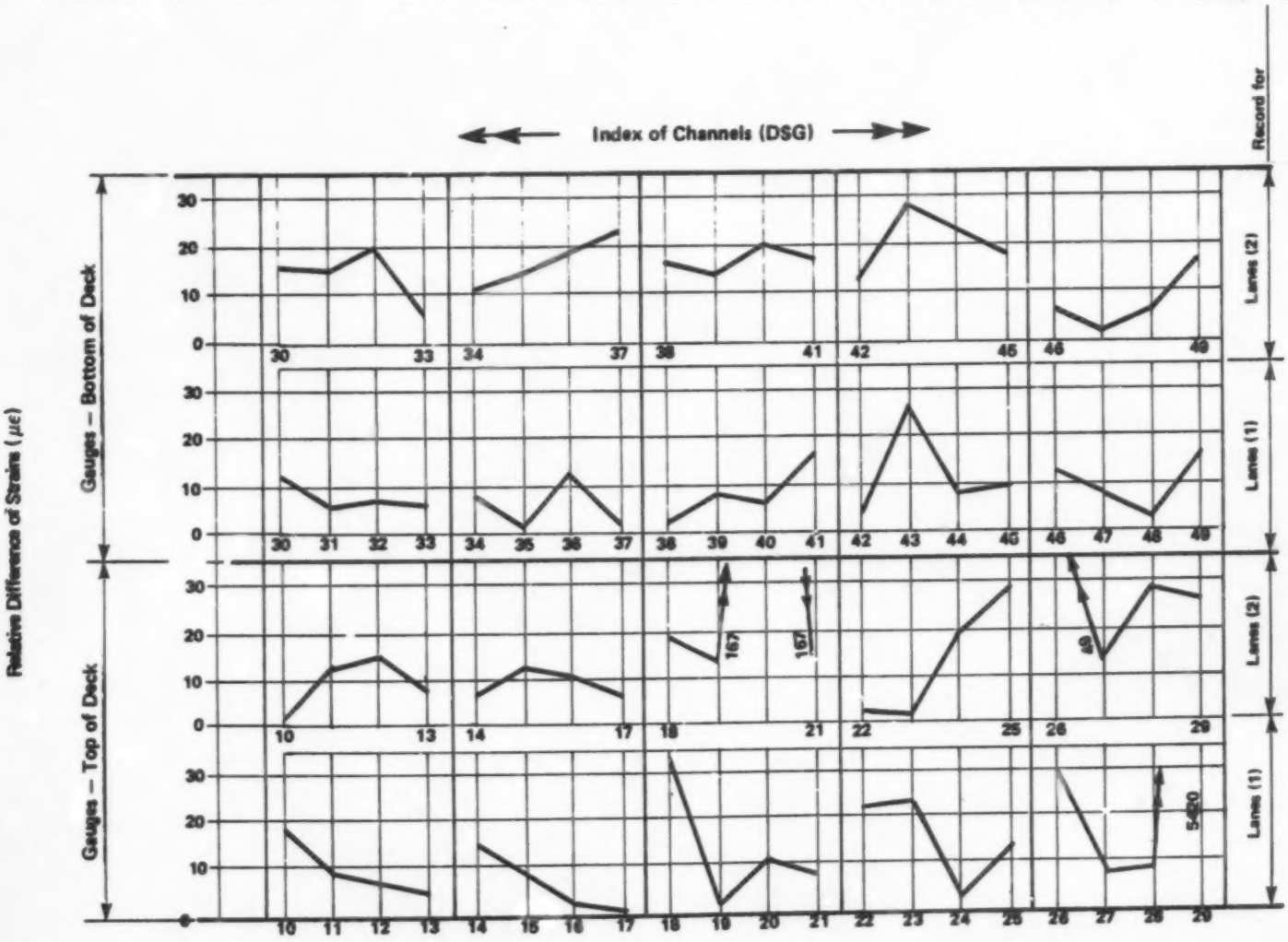

Index of Channels (DSG)
Record for
Relative Difference of Strains (με)
Gauges – Bottom of Deck
Gauges – Top of Deck
Lanes (2)
Lanes (1)
Lanes (2)
Lanes (1)
30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49
10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29
30 20 10 0
167
157
49
5450

Figure 11/ Drift in Gauge Readings Relative Difference Between Pre and Post Load Records